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Organization TC 3600
Bldg/Room CPK5
Examiner: Naoko Slack

Subject: Response to Office Action
Application 09/769,879
Title: Gravity Balance Frame

Dear Examiner:

The word " beam " in claim 1, line 7 is revised to " beams", see attached p.9.

The claims were rejected based on the writing of Chapter 6 of NEHRP (1) on the earthquake resistance system of braced frames, and the prior art of record " Tension-Only Concentrically Braced Steel Frames.."(2). As explained in the original application (3) and later in the "Response to Office Action on October 18,2002" (4), the V-shaped brace and tension-only frames separately are not the distinct features that the present application is to be based upon. The distinct features and novelty of present application are based on combining three conventional components to form an ingenious system, which has never before been conceived. Any one of the three components separately by itself is well known to all structural engineers. However, by putting together all three, they form an innovative, simple and reliable structural system never before applied as described in the original application. These are the V-shaped brace, tension-only brace, and gravity potential energy as discussed in the following:

I. Conventional V-Shaped Brace - This type of braced frame has been used for a long time to resist lateral forces such as earthquake or wind. For earthquake resistance design, the system initially uses the braces to resist both tension and compression forces, and mainly to utilize the strain energy of tension brace beyond yielding (not the gravity potential energy) to resist the earthquake energy once the compression brace buckles. As discussed in Section 1.3.3.1 of Ref. 1, " ... a building will survive by dissipating energy in the yielding of its components...," (see Mark (1) of p. A-1 or p.3 of Ref.1).

As discussed in Section 6.1 of Ref. 1, the V-shaped brace is classified as "Concentrically Braced Frame". For this system, the deflection amplification factor Cd is estimated to be 4.5, (see Mark (2) of p. A-1 and Mark (3) of p. A-2, both from Ref. 1). If plotted by a force-deflection curve, the expected deflection will be 4.5 times the yielding deflection, see Figure A-1 below.

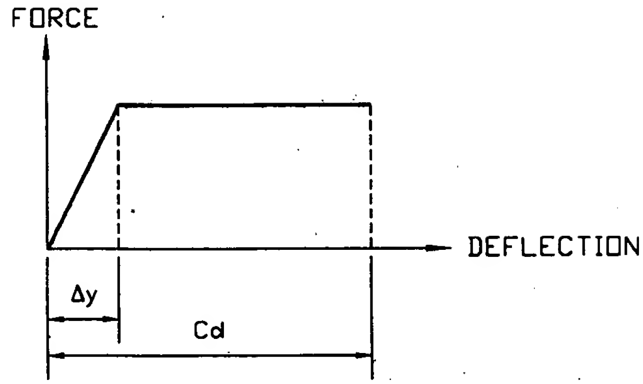

FIGURE A-1

The conventional V-shaped brace system needs to design the beam to carry the gravity load in the bracing bay, (see Mark (4) of p. A-3 or p. 64 of Ref. 1), therefore it is desirable to reduce the gravity load as much as possible. This is not the case in the proposed application where the gravity load is to be carried by the tension-only braces to generate the gravity potential energy (Ref.3), and therefore the heavier the gravity load, the better the system. Also if the beam capacity is exceeded under unexpected larger earthquake forces, the gravity load will cause the conventional system to fail, while the present application is a non-collapsible stable system regardless the earthquake intensity (Ref.3).

Since the conventional V-shaped brace system needs to design the bracing to resist both the tension and compression forces, it requires stocky members with a limited slenderness ratio, (see Mark (5) of p. A-4 or p. 2-250 of Ref. 5), while the current invention needs only very slender tension-only braces. Thus, the present application will save significant amounts of material.

In summary, the conventional V-shaped brace system is different from the present application in the following ways:

1. The Conventional V-shaped brace system uses strain energy of the braces to resist earthquake energy and the beam to support the gravity load, therefore it is desirable to have as little gravity load as possible in the bracing bay. On the contrary, the present application utilizes the gravity potential energy, and therefore the more gravity load in the bracing bay, the better the system.
2. The conventional system uses braces to resist both tension and compression forces and therefore the braces are bulky and stocky, while the present application uses the braces to resist tension only, and therefore the braces are slender, and significant amounts of material are saved.
3. The conventional V-shaped brace will become unstable if the earthquake is larger than the expected design, while the current design will always be stable regardless of the earthquake intensity.

II. Tension-Only Concentrically Braced Frames (TOCBFs): In this conventional system, tension-only braces are arranged diagonally from beam-column joints at the upper floor to lower floor. The strain energy of the tension braces is used to resist the earthquake energy as discussed in the original application (Ref.3). While the present application uses the tension-only braces to induce the gravity potential energy to resist the earthquake energy, and thus create a simple and reliable system.

III. Gravity Potential Energy (GPE): While GPE is a very common concept in basic physics, it has never been utilized in structural systems in a simple and reliable way to resist earthquake or wind

load (Ref.3). In other words, once the structure deforms under lateral load, the gravity load on the girder of bracing bay will move upwards, and thus increasing the potential energy of the system.

The underlying theories of responses to rejection of Claims 1-4 under 35 U.S.C. 112 are addressed above. Also the jointing method has not been specifically claimed, since any conventional method will work for the system as explained in Ref.3, (see 3rd line from the bottom of p. 5 to 1st line of p.6). However, the salient feature of connecting the braces to the center of beam is essential.

Claims 1-6 are rejected under 35U.S.C. 102 based on NEHRP Handbook,1992 (Ref.1).The responses are as discussed above, and summarized below:

1. NEHRP did not disclose "gravity potential energy" being used in earthquake resistant design, rather the conventional systems use strain energy (i.e., yielding of its components).
2. It is desirable to reduce the gravity load in the bracing bay of the conventional V-shaped brace system rather than with "substantial" gravity load, which is actually desirable in the proposed system.
3. The components in the present application may appear to be similar to the conventional V-shaped brace system. However, utilization of gravity potential energy with all conventional components to create an ingenious system makes the present application distinct.

Also rejection of present application is based on the article " Design of Tension-Only Concentrically Braced Steel Frames for Seismic Induced Impact Loading" (2). The difference of the system from the present application is again the utilization of strain energy by the conventional system versus gravity potential energy in the proposed system as discussed above.

In conclusion, the reasons for rejecting the claims based on the prior arts of conventional V-shaped brace system (Ref. 1), and tension-only diagonal brace system (Ref. 2) do not consider the essential different theoretical basis and actual structural behaviors for resisting seismic load as compared to our proposed system. Furthermore these two prior arts have also been discussed in the original application (Ref. 3).

Sincerely

Ma-Chi Chen

Ma-Chi Chen, Ph.D.

REFERENCES

1. Building Seismic Safety Council (BSSC), 1992. NEHRP Handbook for the Seismic Evaluation of Existing Buildings", FEMA –178, published by the Federal Emergency Management Agency, Washington, D.C.
2. Filiatrault, A., and Tremblay, R., 1998. " Design of Tension-Only Concentrically Braced Steel Frames for Seismic Induced Impact Loading", Engineering Structures, 20(12), 1087-1096.
3. Ma-chi Chen, 2001."Patent Application for Gravity Balance Frame", Application 09/769,879.
4. Ma-chi Chen, 2002. " Response to Office Action".
5. International Conference of Building Officials (ICBO), 1997. "Uniform Building Code", Whitter, CA.

Claims: I claim:

1.An earthquake resistant multi-story building utilizing gravity potential energy to absorb earthquake energy, compressing :

(a) a plurality of tension-only braces,

(b) a plurality of beams with substantial gravity load,

(c) a plurality of columns, and

(d) means for jointing the low end of a pair of said braces arranged in a v-shape to the center of one of said beams, and the two upper ends to said columns at each end of said beam, respectively, and repeating the jointing process at predetermined locations,

(e) whereby when said columns move laterally under earthquake motion , said braces will lift said beams upward, and induce gravity potential energy to counter-balance earthquake energy, and afterwards said beams will move downward to their original positions under gravity load, in turn said columns will return to the vertical position due to constraint of said tension-only braces, and thus creating a stable, fail-safe and economical building for resisting earthquake motion.

2. The claim 1 wherein said tension-only brace may be made of steel rod, steel cable, steel wire and any commercially available materials with proper stiffness and strength.

3. The claim 1 wherein said building may be constructed of steel or of reinforced concrete.

4.The claim 1 wherein the joint location from said brace to said beam may be at any point within the span of said beam, preferably at concentrated load to be most efficient.

5.A method of utilizing gravity potential energy to absorb earthquake energy for a

1.3.2 FORCE LEVEL FOR ANALYSIS

Some criteria for the evaluation of existing buildings specify seismic design forces that are lower than those prescribed by the seismic design criteria for new buildings. This concept is generally acceptable because it is believed that building should be substantially below the current standard before triggering the requirement for a costly seismic upgrade and because a higher level of earthquake damage is acceptable in an existing building. This acceptance may be based on an appraisal of the cost benefits of the reduction of damage due to seismic strengthening or on a rationalization that life-safety threats will be reduced even if the possible earthquake damage level may be higher than that anticipated for new buildings.

A reduction is also used in this handbook. The seismic force levels in the 1988 *NEHRP Recommended Provisions* are derived from the recommended normalized response spectra presented in the *Commentary* of the 1988 *NEHRP Recommended Provisions*. For this handbook, the spectral amplification factors are revised to mean values and, thus, the amplification factors of the ground acceleration and velocity used in the 1988 *NEHRP Recommended Provisions* are modified to 85 percent and 67 percent, respectively.

The effect of this modification on an analysis that compares the demand on the structural system due to the probable earthquake to the existing capacity of the structural system can be described by reviewing the concepts on which seismic design requirements are based. The lateral forces prescribed for design of new buildings are used to calculate a required strength. The prescribed forces are not the same as dynamic base shears and connection forces since the basic assumption of seismic design is that the lateral-force-resisting system will have nonlinear displacements if shaken by the design earthquake. Seismic design provisions are based on the concept of an elastic single-degree-of-freedom (SDOF) oscillator responding to random excitation at its base. The concept assumes that the displacement of a nonlinear SDOF system is the same as its elastic counterpart. To change the total displacement, the stiffness, not the strength, of the SDOF must be changed. An elastic strength of the system is required by the 1988 *NEHRP Recommended Provisions* and is determined by use of a seismic design coefficient, C_s, and a response modification factor, R.

If the response modification factor, R, is increased, the nonlinear part of the displacement is increased, not the total displacement of the SDOF system. The total displacement of the SDOF system is determined by the spectral displacement of the prescribed spectra. The 1987 Applied Technology Council (ATC) report that served as the basis for this handbook, *Evaluating the Seismic Resistance of Existing Buildings* (ATC-14), recommends that the spectra for existing buildings be modified to represent mean values of spectral acceleration, velocity, and displacement. Use of the ATC recommended spectra as a basis for the determination of the required strength for existing buildings does not necessarily imply that the ratio of the nonlinear displacement to the linear displacement of the structural system will be increased; rather, use of these spectra and the associated seismic design coefficients implies that the ratio of inelastic displacements to elastic displacements or the ductility is related to the mean probable earthquake instead of to the more conservative probable earthquake.

1.3.3 DEMAND VERSUS CAPACITY

1.3.3.1 Evaluation

One major difference between design and evaluation is point of view. In design, buildings are modeled as elastic systems with stresses proportional to strains. Although actual earthquake forces and deflections may be larger than code forces and calculated elastic deflections, a building will survive by dissipating energy in the yielding of its components if the code provisions concerning force level and detailing have been applied properly. (1) The design lateral forces are obtained from a base shear formula that includes a response modification factor (R), that reflects a trade-off between required strength and ductility; systems with greater ductility qualify for a larger R factor, which leads to lower required forces for design.

Table 2-2 lists the R factors from the 1988 *NEHRP Recommended Provisions* but these factors should not be considered to be ductility factors. The 1988 *NEHRP Recommended Provisions* specifies, in addition to R factors, a deflection amplification factor, C_d. This factor can be used as an estimate of system ductility. The calculated elastic displacements using the required seismic forces are multiplied by the C_d factor to provide an estimate of the total displacements. (2) Part of the difference between the prescribed R and C_d factors is the result

A-2

TABLE 2.4.3.1
Response Coefficients*

R	C_d	System
		Bearing Wall Systems
6.5	4	Light-framed walls with shear panels
4.5	4	Reinforced concrete shear walls
3.5	3	Reinforced masonry shear walls
4	3.5	Concentrically braced frames
1.25	1.25	Unreinforced masonry shear walls
		Building Frame Systems
8	4	Eccentrically braced frames, moment resisting connections at columns away from link
7	4	Eccentrically braced frames, non-moment resisting connections at columns away from link
7	4.5	Light-framed walls with shear panels
(3) 5	4.5	Concentrically braced frames
5.5	5	Reinforced concrete shear walls
4.5	4	Reinforced masonry shear walls
3.5	3	Tension-only braced frames
1.5	1.5	Unreinforced masonry shear walls
		Moment Resisting Frame System
8	5.5	Special moment frames of steel
8	5.5	Special moment frames of reinforced concrete
4	3.5	Intermediate moment frames of reinforced concrete
4.5	4	Ordinary moment frames of steel
2	2	Ordinary moment frames of reinforced concrete
		Dual System with a Special Moment Frame Capable of Resisting at Least 25% of Prescribed Seismic Forces
		Complementary seismic resisting elements
8	4	Eccentrically braced frames, moment resisting connections at columns away from link
7	4	Eccentrically braced frames, non-moment resisting connections at columns away from link
6	5	Concentrically braced frames
8	6.5	Reinforced concrete shear walls
6.5	5.5	Reinforced masonry shear walls
8	5	Wood sheathed shear panels
		Dual System with an Intermediate Moment Frame of Reinforced Concrete or an Ordinary Moment Frame of Steel Capable of Resisting at Least 25% of Prescribed Seismic Forces
		Complementary seismic resisting elements
5	4.5	Concentrically braced frames
6	5	Reinforced concrete shear walls
5	4.5	Reinforced masonry shear walls
7	4.5	Wood sheathed shear panels
		Inverted Pendulum Structures
2.5	2.5	Special moment frames of structural steel
2.5	2.5	Special moment frames of reinforced concrete
1.25	1.25	Ordinary moment frames of structural steel

*The response modification factors, (R), and deflection amplification factors, (C_d), are from Table 3-2 of the 1988 *NEHRP Recommended Provisions*; see these provisions for details.

NOTE: The American Iron and Steel Institute has written a minority opinion concerning this table; see the conclusion of this document.

calculated stress to account for torsion depending on the amount of eccentricity and the distance between braced frames. If the average stress exceeds 30 ksi, a more accurate analysis of the stresses on the bracing elements should be performed.

6.1.2 STIFFNESS OF DIAGONALS: All diagonal elements required to carry compression have Kl/r ratios less than 120.

Diagonal braces can be designed with Kl/r ratios up to 200; however, braces with high Kl/r ratios can buckle at low stress levels and they exhibit poor cyclic performance characteristics. The ductility provided by systems with such light diagonals may not be consistent with the assumed R value (Table 2.4.3.1). The deficiency is in the stiffness of the diagonals.

Procedure: Check the bracing elements and amplify the seismic force by the factor 1.25.

6.1.3 TENSION-ONLY BRACES: Tension-only braces are not used as the primary diagonal bracing elements in structures over two stories in height.

The capacity of tension-only braces may not be adequate for large or tall buildings. Tension braces generally do not perform well. During the first yield cycle, they stretch and then go slack; when tension is applied the next time, it comes on abruptly as a shock that often breaks the brace. If there are only one or two bays of bracing, a further concern is that the building lacks redundancy. The deficiency is in the strength of the braces.

Procedure: Check the braces. (Tension-only bracing of small penthouse structures may not require review.)

6.1.4 CHEVRON BRACING: The bracing system does not include chevron-, V-, or K- braced bays.

Chevron-bracing and V-bracing (Figure 6.1) have a special problem not found in diagonal braces. When one of the braces buckles, large vertical deflections can occur in the floor beam at the joint with the braces. (4) The tension brace tries to drag the beam down while there is little resistance from the compression brace. In this situation, if the beam is in segments that span from the column to the brace point, failure of the braces leads directly to a failure of the gravity load system. The beam should be a single element that can carry the gravity loads without the intermediate support of the braces, but it must be carefully designed to withstand the large forces that the braces can impose on it. Also consider the effect of buckling of a leg of chevron-bracing or V-bracing. Pay careful attention to the continuity, strength, and bracing of the beams and the ability of the connection to permit buckling of the brace while not destroying the capacity for repeated cycles of loading.

K-braced frames (Figure 6.1) also have a special problem. Deformations in the bracing due to tensile yield or buckling can cause lateral deformations of the columns that can cause column buckling or collapse. Current thinking is that K-bracing should not be allowed in buildings of more than two stories.

Procedure: Check all elements in the braced frames. If K-bracing is used in buildings over two stories, amplify the seismic forces in the bracing and columns by the factor $C_d/2$, but not less than 1.5.

6.1.5 CONCENTRIC JOINTS: All the diagonal braces frame into the beam-column joints concentrically.

The concentrically braced frame may have local eccentricities within the joints. It is an important design task to provide for these eccentricities, but the eccentricities must be small enough to limit the action of the members to resisting axial forces with negligible flexural and shearing stresses. If there is a large eccentricity, the frame is a hybrid of braced frame and moment frame. (See Sec. 6.2 for a discussion of frames that are intentionally designed with large eccentricities.) Excessive eccentricity can cause premature yielding of the beams and thereby reduce the strength of the frames. The deficiency is in the strength of the joints.

Procedure: Evaluate the consequence of the eccentricity on the member required to resist it. Evaluate the shear, bending, and axial force requirements at the locations of eccentricities.

P_{uc} = Required axial strength in the column (in compression) ≥ 0

V_n = Nominal strength of the panel zone as determined from Equation 8-1, ksi.

Z_b = Plastic section modulus of a beam, in.^{3}

Z_c = Plastic section modulus of a column, in.^{3}

d_b = Average overall depth of beams framing into the connection, in.

These requirements do not apply in any of the following cases, provided the columns conform to the requirements of Sect. 8.4:

8.6.a. Columns with $P_{uc} < 0.3F_{yc}A_g$.

8.6.b. Columns in any story that has a ratio of design shear strength to design force 50 percent greater than the story above.

8.6.c. Any column not included in the design to resist the required seismic shears, but included in the design to resist axial overturning forces.

8.7. Beam-to-Column Connection Restraint

8.7.a. Restrained Connection:

1. Column flanges at a beam-to-column connection require lateral support only at the level of the top flanges of the beams when a column is shown to remain elastic outside of the panel zone, using one of the following conditions:

   a. Ratios calculated using Eqs. 8-3 or 8A are greater than 1.25.

   b. Column remains elastic when loaded with Load Combination 3-7.

2. When a column cannot be shown to remain elastic outside of the panel zone, the following provisions apply:

   a. The column flanges shall be laterally supported at the levels of both top and bottom beam flanges.

   b. Each column flange lateral support shall be designed for a required strength equal to 2.0 percent of the nominal beam flange strength ($F_y b_f t_f$).

   c. Column flanges shall be laterally supported either directly, or indirectly, by means of the column web or beam flanges.

8.7.b. Unrestrained Connections: A column containing a beam-to-column connection with no lateral support transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral supports as the column height for buckling transverse to the seismic frame and conform to Sect. H of the *Specification* except that:

1. The required column strength shall be determined from the Load Combination 3-5 where E is the least of:

   a. The amplified earthquake force ~~0.4R × E~~ (where the term 0.4R shall be equal to or greater than 1.0).

   b. 125 percent of the frame design strength based on either beam or panel zone design strengths.

2. The L/r for these columns shall not exceed 60.

3. The required column moment transverse to the seismic frame shall include that caused by the beam flange force specified in Sect. 8.7.a.2.b plus the added second order moment due to the resulting column displacement in this direction.

8.8. Lateral Support of Beams

Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral supports shall not exceed 2,500 r_y/F_y. In addition, lateral supports shall be placed at concentrated loads where an analysis indicates a hinge will be formed during inelastic deformations of the SMF.

9. REQUIREMENTS FOR CONCENTRICALLY BRACED (CBF) BUILDINGS

9.1. Scope

Concentrically Braced Frames (CBF) are braced systems whose worklines essentially intersect at points. Minor eccentricities, where the worklines intersect within the width of the bracing members, are acceptable if accounted for in the design. CBF shall have a design strength as provided in the *Specification* to resist the Load Combinations 3-1 through 3-6 as modified by the following added provisions:

9.2. Bracing Members

9.2.a. Slenderness: Bracing members shall have an

$$\frac{L}{r} \leq \frac{720}{\sqrt{F_y}}$$ except as permitted in Sect. 9.5. ← (5)

9.2.b. Compressive Design Strength: The design strength of a bracing member in axial compression shall not exceed $0.8\phi_c P_n$.

9.2.c. Lateral Force Distribution: Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force shall be resisted by tension braces, unless the nominal strength, P_n, of each brace in compression is larger than the required strength, P_u, resulting from the application of the Load Combinations 3-7 or 3-8. A line of bracing, for the purpose of this provision, is defined as a single line or parallel lines whose plan offset is 10 percent or less of the building dimension perpendicular to the line of bracing.

9.2.d. Width-Thickness Ratios: Width-thickness ratios of stiffened and unstiffened compression elements in braces shall comply with Sect. B5 in the *Specification*. Braces shall be compact or non-compact, but not slender (i.e., $\lambda < \lambda_r$). Circular sections shall have an outside diameter to wall thickness ratio not exceeding 1,300/F_y, rectangular tubes shall have a flat-width to wall thickness not exceeding $110/\sqrt{F_y}$, unless the circular section or tube walls are stiffened.

9.2.e. Built-up Member Stitches: For all built-up braces, the first bolted or welded stitch on each side of the midlength of a built up member shall be designed to